ONTARIO WATER RESOURCES COMMISSION

PROFESSIONAL ENGINEERS' GROUP

TECHNICAL SEMINAR No. 2

THEORY & DESIGN OF FINAL SETTLING TANKS

FOR THE

ACTIVATED SLUDGE PROCESS

MR. K. L. MURPHY Ph.D.
McMASTER UNIVERSITY

MR. G. B. CRAWFORD P. Eng.
MR. P. G. EBERLEE P. Eng.
GORE & STORRIE LTD.

JUNE 1965

FOREWORD

The following is a complete summary of the proceedings of the second technical seminar, conducted by the OWRC Professional Engineers Group, entitled "Theory and Design of Final Settling Tanks for the Activated Sludge Process."

Dr. Murphy's address has been printed with his consent and encompasses the first 12 pages of this summary. Mr. Eberlee's address is also printed with his consent and encompasses the next 10 pages of the summary. The remaining 19 pages of the summary include the introduction of the two speakers and the comments raised during the discussion period.

Copies of this summary are being distributed free-of-charge to all those who attended the presentation of the seminar. Any other group members who may wish a copy will be required to pay $1.00 per copy and are asked to contact Mr. P. J. Osmond.

[illegible]
THE
AR[illegible]

CONSIDERATIONS IN THE DESIGN OF SEPARATORS FOR THE ACTIVATED SLUDGE PROCESS

by

K. L. Murphy,

Faculty of Engineering,

McMaster University.

The activated sludge process in which dissolved and colloidal organic matter is removed from a liquid waste through the growth of aerobic microorganisms is a two step process involving:

1. growth of microorganisms, and
2. separation of the suspended microorganisms from the carriage liquor.

All to frequently "failure" of the process, as judged by the inability to obtain specified objectives, is caused by poor separation of the microorganisms rather than the attainment of microbial growth.

This may best be visualized by a consideration of a hypothetical idealized system presented in figure 1 operating at a steady state.

The solids concentration in the aerator, assuming perfect mixing, can be obtained by a mass balance about this unit:

$$c_i q_i \quad + \quad c_u q_r \quad - \quad c(q_i + q_r) + V_a \frac{dc}{dt} = 0$$

where: $\frac{dc}{dt}$ = unit production of sludge.

letting: $r = q_r/q_i$ = recycle ratio,

$\bar{t}_a = V_a/q_i$ = mean solids residence time in aerator based on influent flow, and

$c_i = 0.$

then: $c = \frac{1}{1 + r} \quad (r\, c_u + t_a \frac{dc}{dt}).$

It is apparent, for a given system, with a given organic loading, that the solids concentration in the aerator is dependent upon the recycle ratio together with the underflow solids concentration. It follows that an increase in underflow solids concentration would be a more effective means of increasing the aerator solids concentration as it does not shorten the time for sludge growth - the effective time for sludge growth in the aerator being $\bar{t}_a/1 + r$.

One task of the final clarifier is to provide adequate amounts of return sludge to keep the desired solid concentration in the aerator. An overall mass balance of the aerator and clarifier system indicates:

$$c_i q_i - c_e q_e - c_u q_w + (V_a + V_s) \frac{dc}{dt} = 0$$

which when : $q_e = q_i - q_w$,

$\frac{q_w}{q_i} = w$ = wastage ratio,

$\bar{t}_{a+s} = \frac{V_a + V_s}{q_i}$ = mean solids residence time in system, and

$c_i = 0,$

becomes: $\bar{t}_{a+s} \cdot \frac{dc}{dt} = w\, c_u + (1 - w)\, c_e.$

The maximum effluent solids concentration that will permit steady state operation will occur when the wastage is reduced to zero thus:

$$c_e \max = (\bar{t}_{a+s} \cdot \frac{dc}{dt}) .$$

The secondary clarifier must provide for a separation of solids so that effluent solids concentration will not exceed the production of solids in the system. From this it may be concluded that secondary clarifier design must be based upon:

1. clarification or the ability to adequately separate the solids from the liquid to prevent "wash-out", or maintain desired effluent standards, and
2. to supply the mass of recycle sludge at a "maximum" concentration thus minimizing aerator volume for effective dissolved and colloidal organic removal and subsequent growth.

The laboratory approach to the design of separation facilities for dilute suspensions of discrete or flocculent particles, using a long tube settling column, has been given by Fitch (1957) and O'Connor and Ecknefelder (1958). This technique is not applicable to the design of secondary clarifiers as the influent has a high solids concentration, generally designated as a class III suspension (figure 2). The settling velocity of such solids is governed not by the Stokes' Law relationship for individual particles but rather by the properties of the suspended mass. A distinct interface is formed, the settling velocity of which is a function of the particle concentration.

This may be observed in the laboratory with a batch settling test (figure 3a). Initially a uniform concentration exist, (B). As settling progresses a zone of clarified liquid develops (A) at a rate which is proportional to the original concentration, as all particles in B are settling at a uniform rate. Concurrent with the formation of the clarified zone, solids are concentrated to a point where they are mechanically supported by solids below (D). Immediately above this compression zone is a zone (C) in which the concentration solids varies from that of the indered settling zone to that of the compression zone. Figure 3b presents a picture of the temporal position of a particle initially located on the surface, which in the case of concentrated suspension also represents the temporal position of the interface.

In the consideration of concentrated suspensions two factors must be considered:

1. the clarification capacity which corresponds to the initial settling rate of the interface (u), and
2. the thickening capacity or solids loading rate.

The thickener area (A) required for a given clarification capacity is obtained from:

$$A = q / u$$

where: q = influent volumetric flow.

The solids loading rate is defined as:

$$\frac{q\,c}{A} = \frac{c\,Z}{t_u}$$

where: t_u = time to attain underflow concentration

Z = initial height of interface

Values of t_u can be obtained graphically from plots of batch settling data (figure 4). A straight line is drawn tangent to the compression point (c_2) which is normally located on the bisector of angle formed by the hindered settling and compression zone tangents. The underflow concentration is given by the mass balance:

$$c\,Z = c_u Z_u$$

or

$$c_u = c\,\frac{Z}{Z_u}$$

where: c_u = underflow concentration

Z_u = height of the interface if all the solids in the system were at the underflow concentration

According to Eckenfelder and Mancini (1965) the sludge depth can be approximated by a consideration of the average time and concentration in the compression zone. The time that the sludge is in the compression zone in the thickener is $t_u - t$ with a concentration range of $c - c_u$.

The solids loading in this interval is:

$$\frac{q\,c\,(t_u - t)}{c}$$

This must be equal to the depth of the compression zone (H) times the thickener area (A) giving:

$$H = \frac{q\,c}{A} \cdot \frac{t_u - t}{c_a}$$

It must be noted that we are dealing with the settling velocity of the interface and that the time intervals obtained in laboratory apparently should be adjusted to compensate for the variation in depth from the test cylinder to the prototype thickener.

This analysis presupposes that velocity is a function of concentration. Where there is doubt about the efficacy of the results the method of Coe and Clevenger described by Behn and Liebman (1965) may be applied. This method determines experimentally the settling velocity (U_1) for a given layer concentration (c_1). The required thickener area is given by the maximum A obtained from the various pairs of U_1 and c_1 in the expression:

$$A = \frac{q\,c_u}{U_1}\left(\frac{1}{c_1} - \frac{1}{c_u}\right).$$

They make use of a sludge compression formula to evaluate the depth requirement.

The underflow concentrations obtained from a continuous gravity thickener will vary with the depth of the sludge blanket, reflecting a time dependent phenomena. Increasing the detention time in a unit will generally result in higher underflow solids concentrations. According to Eckenfelder and Mancini (1965) providing the additional detention by increasing the area (i.e.

reducing the solids loading) results in greater improvement in thickener performance then by increasing blanket depth particularly with biological sludges. Little or no information is available on process influence on clarification effectiveness. Poor hydraulic characteristics can cause scour and carry-over of solids decreasing expected results. Maximum efficiency for a given sludge can be obtained from solids analyses of the supernatant following laboratory testing.

Several limitations of laboratory testing make extrapolation directly from laboratory to prototype units subject to the usual risks associated with engineering extrapolations. In the batch test there is no simulation of underflow. When underflow is a significant fraction of the incoming flow a conservative areal estimate will result. This may in turn be more than compensated by hydraulic conditions, variations in influent solids concentration and agitation of the compacted sludge. The effect of mixing in batch studies is illustrated in figure 5. The simulation of the collector mechanism influenced both clarification and compaction zones. Mancini (1962) detected no influence of cylinder diameter on experimental results but found a dramatic influence of depth on the area required for a given compaction. No variation on clarification capacity, initial settling rates, was reported. Subsequently Eckenfelder and Mancini (1965) reported the use of a 8 foot column replacing the former 1 litre cylinders.

The benefits of increased sludge holding time in increasing underflow have been mentioned. With biological sludges a limit

on sludge retention may exist. Delay in removing sludge from final settling basins and thus subjecting it to periods of prolonged anaerobiosis is generally considered to be harmful to the process. Considerable money and time have been expended to ensure minimum anaerobic storage. Reaeration facilities have been included to counteract further the effects of anaerobic conditions. In general, these effects may be summarized as:

1. rising or floating sludge caused possibly by denitrification,
2. increased oxygen requirements when the sludge was returned to the aerator, and
3. decreased assimilation capacity by the microorganisms.

Recently, three independent studies by Wuhrmann (1963), Westgarth (1963) and Murphy and McLellan (1965) have discounted the latter two effects for normal activated sludge operation. Wuhrmann in the laboratory subjected activated sludge to anaerobic conditions for periods up to 4.5 hours and could not detent any significant change in respiration rate. Westgarth operated two pilot plants in parallel one of which incorporated an unaerated sludge holding tank. No significant differences in effluent 5 day BOD and suspended solids, or oxygen uptake of the sludge were observed. Table 1 (Murphy and McLellan, 1965) summarizes the results of a laboratory study on the effect of a three hour and a six hour anerobic storage period on the ability of the sludge to utilize soluble organic matter. With the three hour storage approximately 70-71 percent of the added C.O.D. (Chemical Oxygen

Demand) was removed in one hour and 82 percent in the first three hours of subsequent aeration. Similarly during the first three hours following a six hour storage 80 percent of added COD was removed by both aerobically and anaerobically stored sludges. The total COD removed and amount of oxygen utilized per unit mass of cells to remove a unit of organic matter was essentially the same following both times and both types of storage. This indicates that subsequent removal of organic matter and oxygen utilization was independent of the type and length of storage up to six hours. These three investigations do not rule out entirely the possibility of harmful effects from anaerobiosis for a particular sludge.

The problem of rising sludge may still limit solids detention in final clarifiers. Eckenfelder and Melbinger (1957) have indicated that the time required for gasification and floating was intimately associated with aeration time-extended aeration periods decreasing the likelihood of trouble. They have indicated the desirability of testing to determine maximum retention periods. Little data is available on actual sludge detention times within clarifiers but a number of factors undoubtedly influence the retention period. These include:

1. Basin hydraulics,
    a) terminal conditions,
    b) water flow patterns,
    c) underflow flow patterns,
    d) density currents, and
    e) short circuiting and turbulence.

2. Solids settling rate.
3. Sludge collection,
    a) type, design and speed,
    b) sludge withdrawal rate,
    c) liquid depth, and
    d) solids characteristics.

With the basin hydraulics the existance of a sizable underflow complicates analysis as few if any hydraulic studies have included both the complicating factors of solids and underflow. If the sludge blanket offers sufficient hydraulic resistance the underflow will be distributed over the entire basin. When channeling, with localized flow, occurs the entire basin hydraulics will be altered and a drastic decrease in underflow concentration will occur. Anderson (1945) and others have reported the existence of density currents in final clarifiers. Similar effects have been observed in clarifiers where no density differentials are probably the result of the residual momentum of the influent after entering the basin.

Inlet design should provide for destruction of momentum, rather that simply redirecting the incoming flow, if the hydraulic conditions assumed in the laboratory tests are to be achieved. The flow entering the basin should have a moderately low velocity and be distributed uniformly across the clarification zone. Conventional inlets do not accomplish this distribution and long narrow basins have been employed to compensate for inefficient inlet devices. Simple pipe inlets accomplish neither horizontal

nor vertical distribution; full width weirs provide a horizontal distribution but if "free falling" cause bottom-flows or if submerged top-flows. Solid baffles have been used to modify inlet structures with the result that the flow is redirected. The use of vertical slotted baffles has improved the hydraulic characteristics as measured by dispersion testing (table II and figure 6, Murphy, 1962). A number of other patented Inlet forms show similar improvement including "Stengle jet", a Geiger tube" and tangential raceway types.

Where density or momentum currents exist it is beneficial not to locate the outlet on the opposite wall or point of upturn of the density current. Upon relocation of the outlet weir Anderson (1945) among others noticed improved performance while improved basin hydraulics as measured by dispersion testing is evidenced in table 2 and figure 6 (Murphy, 1962).

Much of the information needed for a rigorous design of a final clarifier is at present no available. Since this unit plays a critical role in biological waste treatment it would seem logical that further investigation should be attempted. A successful final basin must produce excellent clarification or separation of solid and liquid phases while at the same time provide a concentrated underflow for return to the aerator. While these functions may not be mutually exclusive in an "ideal" separation, with the normal operating disturbances encountered in actual clarifiers attainment of both appears difficult.

References

1. Anderson, N. E., "Design of Final Settling Tanks for Activated Sludge," Sewage Works Jour. 17, 50, (1945).
2. Behn, V. C. and Liebman, J. C., "Analysis of Thickener Operation," Proc. of the Sanitary Engr. Div. ASCE, 89, SA 3, 1, (1963).
3. Eckenfelder, W. W. and Mancini, J. L., "Thickening of Industrial Sludges", Proc. 5th Texas Ind. Water & Waste Conf., 64, (1965).
4. Eckenfelder, W. W. and Melbinger, N., "Settling and Compaction Characteristics of Biological Sludges, 1. General Considerations, "Sewage and Ind. Wastes, 29, 1114, (1957).
5. Fitch, E. B., "The Significance of Detention in Sedimentation," Sewage and Ind. Wastes, 29, 1123, (1957).
6. Mancini, J. L., "Gravity Clarifier and Thickener Design", Proc. 17th Purdue Ind. Waste Conf., 267, (1962).
7. Murphy, K. L. "Hydraulic Tests of Sedimentation Basins", Proc. 9th Ontario Ind. Waste Conf., 271, (1962).
8. Murphy, K. L.,"Tracer Studies in Circular Sedimentation Basins" Proc. 18th Purdue Ind. Waste Conf., 374, (1963).
9. Murphy, K. L. and McLellan, J. C., "Some Effects of Anaerobic Storage on Activated Sludge" Proc. 5th Texas Ind. Water and Waste Conf., 194, (1965).
10. O'Connor, D. J. and Eckenfelder, W. W., "Evaluation of Laboratory Settling Data for Process Design., "Biological Treatment of Sewage and Ind. Wastes, Vol. 2., Edit. by McCabe and Eckenfelder, Reinhold, New York, (1958).
11. Talmage, W. P. and Fitch, E. B., "Determining Thickener Areas", Ind. and Engr. Che., 47, 38, (1955).
12. Westgarth, W. C., "Anaerobiosis in the Activated Sludge Process", Ph.D. Thesis, Univ. of North Carolina, (1963).
13. Wuhrmann, K., "Effect of Oxygen Tension on Biochemical Reactions in Sewage Purification Plants, "Advances in Biological Waste Treatment, Edit. by Eckenfelder and McCable, Pergamon Press, New York, (1963).

FINAL SETTLING TANKS FOR THE ACTIVATED SLUDGE PROCESS

The activated sludge process currently enjoys high standing among methods of sewage treatment and it has received considerable and intensive research and appraisal in recent years. The evolution of the process is described briefly, but well in the Journal of the Water Pollution Control Federation for February, 1965. We are reminded that its general use is no older than most of us here tonight so it is not surprising that much of the information gained from research and observation is still quite new.

It is apparent from the literature that the greater part of the research and development activity has been concerned with the biological phase of the activated sludge process. Research data or other information on the final settling tank is not as well covered in the publications. This is not fair reflection of the relative importance of the final settling tank which could be considered, from the design point of view, as the most important element of the process. For instance, conditions in the aeration tanks can be altered at relatively little cost and short down time by adjusting the amount of air applied or the type of air diffuser used. This cannot be said of the final settling tank, which once built is not easily altered. Furthermore, the final settling tank traditionally consists of a concrete structure in which the liquid contents are perhaps little influenced by the readily movable devices, such as baffles, inlet ports, weirs and other features. The final settling tank is a hydraulic structure and the successful designer must recognize the particular characteristics of mixed liquor and arrange his tank to suit. He will have

little success if he attempts to make the mixed liquor behave in a tank to which it is not suited. It will find its own way to behave and that may produce unfortunate results. To make the problem more difficult, the nature of sewage reaching treatment plants is known to vary considerably from one community to another. This is not going to become less so with the increasing complexity of our modern society. Thus, every case must be given individual treatment.

Basically, the final settling tank has two functions to perform. First, the tank must collect the mixed liquor solids for return to the aeration unit within a reasonably short period. In the second place, the tank is expected to draw off a well clarified and sparkling effluent from the plant for discharge to the receiving waters.

The behaviour of the type of solids in the final settling tank must be considered as distinctly different from the behaviour of solids in primary settling tanks. The settling of solids in the primary tank is analogous to the settling of grit in straight line hydraulic grit channels. That is, the length, depth and detention time may all be reasonably co-ordinated to achieve settling of discrete particles of a given size and density.

In the final tank, use of the word "settling" may actually be misleading as a description of the concentration of activated sludge solids. Mr. Norval Anderson of Chicago was perhaps the first to discover and report on the phenomenon of sludge density currents in final tanks. His discovery of this characteristic at the

Southwest Plant in Chicago induced him to make subsequent studies at the Chicago north side plant and at Columbus, Cleveland and New York. In both rectangular and circular tanks, he found well established sludge density currents, while in observations made on primary settling tanks, no such currents could be found.

Mr. R. H. Gould of New York City has also reported on the density current phenomenon and his successor, Mr. Wilbur Torpey, has exploited the density current theory in the design of tanks in the New York area. In our work, the final settling tanks at the Metropolitan Toronto main treatment plant, the largest in Canada, were designed distinctly on the sludge density current theory; and subsequent velocity measurements in the tanks in actual operation have proven the existence and certain characteristics of a sludge density current.

A density current is in effect the concentration of the activated sludge solids immediately after inlet into a settling tank. The solids, because of greater density, fall quickly to the bottom of the tank and flow along the bottom of the tank away from the inlet. Wilbur Torpey, who has a reasonable rule of thumb for most aspects of sewage treatment, suggests that a sludge density current has a velocity of 5 feet per minute for each 1,000 parts per million of mixed liquor solids. At the Metropolitan Toronto main treatment plant, this theory was indicated fairly reliably, in that mixed liquor solids of 2500 to 3000 parts per million resulted in a measured density current velocity of 10 to 15 feet per minute near the tank bottom.

It may be well to note here that in our experience, our company has traditionally shown a preference for rectangular straight line final tanks. We have had little experience with circular clarifiers. It is our opinion, however, that successful final clarifiers of either type may be constructed providing the tank details consider the nature and behaviour of the sludge density current, the collection of sludge therefrom and the location and nature of effluent weirs. Traditionally, we have experienced high degrees of clarification with the rectangular type clarifiers. Dr. Clair Sawyer, recently of M.I.T. and now with Metcalf and Eddy of Boston has, in his writings, indicated some deficiencies of poorly designed circular clarifiers.

In view of the foregoing discussion, what are the preferable design parameters for final settling tanks? The sludge density current theory requires that the tank provide near its inlet for the assembly of the sludge solids for prompt draw-off. The point of draw-off must be so located that the solids may be removed at reasonable consistency and so that the solids have a short duration period in the final tank in order to avoid suffocation and death of the aerobic organisms. Beyond the draw-off point, the tank must provide sufficient capacity for clarification of the effluent before it is removed at the effluent weir. Briefly, therefore, the final tank design should accomplish short detention of solids and long detention of effluent.

In order to compensate for varying raw sewage and mixed liquor strengths, for varying temperatures, for varying efficiencies of plant performance due in part to the time in which

adequate testing or sampling may be accomplished, and for other variables, a detention time of three hours on an average flow basis is usually recommended. We believe the surface overflow rate should not exceed 800 U.S. gallons per square foot per day on an average flow basis, and should never exceed 1200 to 1300 U.S. gallons per square foot per day.

Probably the variance of tank depth is not too critical within reasonable limits. Providing other criteria are satisfied, any tank depth of 10 to 15 feet is acceptable and it is usually in this range that the tank structures are most economically built.

It may be questionable at this time for anyone to define or describe the optimum arrangement of tank inlets, effluent weirs and sludge draw-off points. We have constructed rectangular final tanks with the sludge draw-off point at the inlet end, at the outlet end close to the effluent weirs, and also at mid-length. Since each general type of tank has performed very efficiently, it must appear that the most important of the criteria recited here remain unknown.

If the sludge density current theory applies to all final tanks, then the current must assemble very quickly if a tank with sludge draw-off point immediately at the inlet can be successful. This may well be and Sawyer has, in fact, described the manner in which the sludge solids enter and immediately drop to the bottom of a final clarifier as comparable to a waterfall.

In a tank with sludge draw-off at the end remote from the inlets, the sludge density current theory would appear to be used to advantage, on the premise that the density current would continue along the bottom to the far end of the tank increasing

in concentration, and the clarifying effluent would roll back on the surface toward effluent weirs at mid-length or even closer to the inlet end.

From results in plants where each of these types of final tanks is employed and where the total removals are consistently in excess of 90 percent, we hesitate to state that either type has a definite advantage over the other.

At the Metropolitan Toronto main treatment plant at Ashbridge's Bay, the final tanks are 300 feet long with the sludge draw-off point at mid-length. As noted before, the sludge density current has been measured at 10 to 15 feet per minute. In addition, dye tests have shown the detention time to be no more than 20 minutes for some effluent arriving at the first effluent weir. These tanks have consistently indicated a sufficiently fast sludge return rate, and through most seasons of the year, a high overall removal is accomplished. These tanks have only two hours' detention at average flow rate; however, during cold winter months, some deterioration of effluent quality is observed. In large size tanks, such as those at Ashbridge's Bay where considerable length of tank is almost a necessity, we believe the mid-length draw-off point is a good design feature, aimed of course at obtaining quick return of the activated sludge.

Tank inlets may be of many varieties but in general they should provide reasonably uniform and symmetrical distribution of the mixed liquor to the final tanks. It is common practice, in small and medium size tanks, to arrange inlets in shallow open channels at the tank surface.

Submerged inlets have also been employed, usually in small tanks where the mixed liquor is piped in and the inlets are turned towards the inlet end wall for baffling. Inlet velocity should probably be in the range of 1 to 2 feet per second or as slow as is feasible without settling of solids in the inlet channels.

Traditionally, final tank overflow weirs are constructed of sufficient length that the overflow rate per foot is low, perhaps 5,000 gallons per day per foot of length. There are several tanks in New York City plants, however, where a single weir is employed near the end of rectangular-type final tanks and having an overflow rate of about 100,000 gallons per day per foot. We have observed there an overflow crest 1 to 1½ inches in depth. The effluent, when viewed, could be described as clear and sparkling. Such phenomena could appear to indicate that if a design takes into account the criteria described earlier, the effluent weir length need not be as long as is commonly employed.

Effluent weirs are commonly fabricated of thin steel or aluminum plate. Many weirs are flat crested but recent design has often included V-notch weir plates with the V-notches at approximately 12-inch centres. Such design allows for uniform collection of effluent over the weir length especially where a low overflow rate is used and it may be difficult to maintain the entire length of weir at exactly the same elevation.

Final tank clarifying mechanisms of the chain pulled wood

flight type are economically constructed in widths of about 12 to 18 feet. In our experience, 16-foot long flights are commonly employed. Division walls between multiple passes in a single tank may be so-called dwarf walls with sufficient concrete structure to carry the flight and chain mechanism.

As you are aware, in this type of structure, it is quite feasible to arrange for surface scum collection with the return travel of the sludge scraper flights and this feature is often provided in the tank design. The need for scum removal facilities has been strikingly demonstrated where they have been incorporated into plant extensions alongside tanks with no scum collection. The amount of scum removed is surprising even with apparently efficient removal in the primary tanks. In this connection, placement of the effluent weirs in the mid portion of the tank to avoid the upturn of the density current at the end remote from the inlets tends to cut down on scum removal efficiency. Scum will form on the surface beyond the effluent weirs with no means of entrapment and it will pass over into the effluent.

A brief comment on sludge collection equipment may be in order. Assuming that the sludge density current attains a velocity of 10 to 15 feet per minute, the function of the scraper mechanism operating at 1 to 2 feet per minute on the tank floor may be questioned. In the rectangular straight line tank, the chain pulled wood flight scraper must obviously be moving with or directly against the travel of the sludge

density current. In the circular tank, the typical sludge collector rotates and may be assumed to plow the sludge solids row by row towards the centre draw-off point. The blades actually travel transversely to the direction of a sludge density current and again, the velocity is much less than that of the current. In either case, it is indicated that the real function of the collector is to prevent possible adherence of sludge solids to the tank bottom, where the activated solids would obviously die and an operating problem would evolve. At the Metropolitan Toronto main plant, for some days, the final tank collectors were operated only certain hours each shift and no deterioration of process could be observed. There would seem to be little justification for continuous running of the collecting mechanisms and we suggest this matter might be studied at any of your plants. Again, however, we suggest to you that each plant will have its own characteristic sludge and we know you need little warning that the operation you find successful at one plant may not necessarily be duplicated at another.

Another comment may be made on clarifying equipment. The replacement of wearing shoes, chains and flights is a costly matter and replacement of various elements of the mechanism may be necesary at frequencies, depending on the use, of 2 to 15 years. While some replacement parts, such as flights and wearing shoes are not unduly costly in themselves, the tediousness and cost of making the replacement together with the

downtime of the tank required, make obligatory employment of the best possible equipment. To this end, we feel that the manufacturers should be encouraged by rigid specifications to continuously upgrade the quality of collector equipment. Certain new products are now on the market including plastic flights, higher strength chain materials and so on.

We are currently studying means of increasing the life of wearing shoes in contact with tank bottom rails. While the traditional tank bottom rail is a standard A.S.C.E. section with convex flange we have installed at one plant, plastic wearing shoes and flat flange rails. It is believed that the added area of contact between the plastic wearing shoe and the rail will provide a lesser rate of wear and considerably increased shoe life. Observations made to date indicate that this will be so.

These are our comments, gentlemen, on current design of final settling tanks used with the activated sludge process. If we seem to be uncertain as to the optimum design for any detail of the perfect tank, it should be considered as further indication of design improvements yet to be achieved. That so many existing final tanks with varying design arrangements are so successful may be more to the credit of the activated sludge process than to the designers themselves. It is our hope that our remarks will at least engender further curiosity and the will for research and experiment toward development of further improved final settling tanks.

A. Townshend

We wish to welcome you to our second annual technical seminar. We have some guests with us this evening. We extended an invitation to technical members of staff in the Commission other than engineers, and we also have two representatives from Metropolitan Toronto that I would like to call on George Trewin to introduce to us.

G. Trewin

I am pleased this evening to be able to welcome to our discussion two experts from Metro. One of them is responsible for the operation of the Humber Sewage Treatment Plant and one for the main sewage treatment plant. I believe they will be able to add something to our discussions tonight and without further ado I'll welcome Mr. Ted Kruk, Mr. Humber, and Mr. Main Plant, Mr. Earl Boldock. Thanks very much.

A. Townshend

Thank you very much, George. Last year at our first seminar our topic was the Theory, Design and Construction of Waste Stabilization Ponds. Fortunately, I find myself in a position to tell you that these are now published and available. In our next Newsletter, we will be giving you a breakdown on the printing of them and the cost to the group. There has been a slight deficit and we need to sell about twenty copies to break even on it. So I would like you to make the other engineers in the group aware of this bulletin. It is available to members of staff in the Commission. For those that have read it, you will realize that there was considerable difficulty with the

recording of the discussion period. First of all, when we do break off for discussion, I would like each and every one of you to remember to come up to the microphone in the centre and speak into it clearly. I have provided all of you with small blue cards, which I hope you will use to jot down notes and thus formulate your thoughts in your mind ahead of time. I think this will facilitate our having better presentation during the discussion period. One of the purposes of the seminar is to give all members of our group an opportunity to speak in this atmosphere in a technical discussion and, as you know, especially in Ontario a lot of our technical conferences seem to hinge around the discussion period, and this is an opportunity for you, by practicing here, to better equip yourselves for taking part in the discussion group at conferences. Our format for this evening will be as follows: We will have one talk on the theory of settling tanks, which will be followed by a second talk on the practical aspects of design. Then we will break for coffee and doughnuts in the luncheon hall and then come back for the informal discussion period.

I would now like to introduce to you our first speaker this evening. Dr. Keith Murphy was raised in Toronto, graduated from the University of Toronto in 1954 and completed his doctoral work in Wisconsin in 1961. He has returned to Ontario and is now Associate Professor, Civil Engineering, McMaster University. Most of you know Dr. Murphy. He did much of his doctoral work on the hydraulic aspects of settling tanks and now, at the

University of McMaster has given leadership in the study of the effects of holding sludge in the return sludge system. So, with these comments, I would like to introduce to you, Dr. Keith Murphy.

Dr. Murphy - See speech.

A. Townshend

Thank you very much, Dr. Murphy. To complement the information which Dr. Murphy has given us, we have invited representatives from Gore & Storrie, one of our prominent consulting engineering firms in the province. We have with us George Crawford, who is also a graduate from the University of Toronto, who following the war joined the firm of Gore & Storrie, was appointed Director in 1958, and has been very closely associated with the Main Sewage Treatment Plant in Metro. This has been his baby and he lives with it day by day. For those of you in Plant Operations, you will know he has also been associated with our Lakeview Sewage Treatment Plant. Mr. Crawford will show us some slides and will take part in the discussion.

Also with us this evening from Gore & Storrie is our speaker, Mr. Peter Eberley. He was born and raised in Lindsay and came to Toronto to attain his bachelor's degree in Civil Engineering in 1956. He is a project engineer with Gore & Storrie and, as such, has worked on the design of a number of sewage plants including the Barrie sewage plant, Collingwood's primary plant and Brockville's primary plant. More recently,

he has been working on water filtration plants. So, I would now like to call upon Peter Eberley to give us some of the experiences of Gore & Storrie in the design of final settling tanks.

Mr. Peter Eberley - See speech

DISCUSSION (EDITED)

A. Townshend

In a discussion of the two papers, I think in fairness to those who made the presentation that we should give them an opportunity to speak first. I would like to begin by calling on Dr. Murphy for a few comments.

Dr. Murphy

I would like to start off by saying that I agree violently with what the other speaker said. I can't find anything with which I am in disagreement. There certainly are momentum currents in secondary clarifiers and, if you look for them in primary clarifiers or in pure water clarifiers, with those type of inlets, you will find them too. Furthermore, on a rough check, I calculated those velocities to be about 10 to 15 feet a minute and these are in the ball park with what we've found in a six-foot diameter basin with pure water. They check out pretty closely. So there is no doubt that we find the velocity profile with the conventional type of inlet or with a solid baffle in front of the inlet almost invariably indicates a very high bottom velocity. The only difference being the order of

magnitude - 7 to 15 feet are the most commonly quoted numbers. I think that about sums up my comments; my only question then is, are these things really density currents or are they momentum currents. I think that for the most part they are probably momentum currents aided by the settlement of the solids. Certainly, once you get the current down there, it will behave the same no matter whether it's a density or momentum current.

G. Crawford

Well, I haven't very many written comments. In fact, I haven't any written comments. I enjoyed Dr. Murphy's paper immensely. My sanitary engineering post-graduate work is not very great and it's a pleasure to hear someone who has his thoughts correlated in such an educated way and can put down complicated formulae. It scares you to death in the first place and he made it look awfully simple. I'm the last one to say that every final clarifier will have a sludge density current of the nature which I have experienced. By way of interest, the one at the Ashbridges Bay plant was measured with a velocity meter and had velocities up to 15 feet a minute in the bottom and 30 inches of a 15-foot depth. Whether they are velocity or momentum currents they certainly went to the bottom of the tank in a hurry. We found zero velocity about 30 inches or three feet off the bottom of the tank and we couldn't measure the velocity at the surface. However, observation of particles indicated a slow movement towards the front or inlet end of the tank. The scum collector moves in that direction also and undoubtedly has some influence. Since there has to be some flow toward the inlet to provide turnover, density currents or momentum currents

must be present in every tank. However, at the Guelph and Peterborough plants we were unable to find a current. The Peterborough tank is a conventional rectangular tank with the inlet and the return sludge draw-off hopper at the same end of the tank and with the effluent weirs at the far end. The Peterborough tanks worked very well. The Guelph tanks are about the opposite to the ones in Peterborough. The draw-off point is at the outlet end, therefore the density current should come in, drop down to the floor and go along to the outlet. Here again we were unable to measure a current so don't ask me just why one behaves a little differently than another.

Again, the one interesting point is that if you have a current of from 5 to 15 feet per minute surely the flights moving at one foot a minute are not going to help in removing the sludge. We should therefore be able to prolong the life of the mechanism by running it periodically.

A. Townshend

Thank you, Mr. Crawford. Before I throw the discussion open to our own staff, I wish to call upon Mr. Kruk. As you know, the Humber plant has very large circular Infilco clarifiers and perhaps Mr. Kruk can offer some comments concerning the need to return sludge quickly and the carrying of a sludge blanket.

Mr. T. Kruk - Humber Plant (Metro)

My name is Ted Kruk and I have been in charge of the Humber plant since its inception. I can speak from practical experience now on some of the things that have caused us trouble and how

we have overcome them. I would like to later ask the consultants and Dr. Murphy a question or two.

We have 90-foot diameter, circular tanks and when we first started operating I used a low rate of return. This, to my mind, was the best way to operate these large tanks as this provided a more concentrated sludge to return to the primaries and it also increases your capacity throughout the whole system. The time the sludge stays in the finals is, of course, the time that it takes for it to settle out and be scraped into the outlet hopper. Now, in my mind, this is very vital and I don't think tanks should exceed 95 feet in diameter, at least for the type of sludge that we have at the Humber- a very sensitive type of sludge. We had to change from the conventional activated sludge process to the step aeration process to obtain a non-degradable or stable type of sludge in the final tanks for the period of time that we need. When I went away on vacation, someone else took over and they started increasing the rate of return from the finals, as much as they could. This resulted in the formation of a hole in the sludge around the hopper. Sludge beyond one-third the distance from the hopper to the periphery was not removed quickly enough and it became septic and contaminated the other sludge in the tank. You can't transfer this by ordinary methods that the design engineer leaves for the operator, i.e. by opening the valve at the side of the tank. So my experience has been that at the lower rate of return, you get a more concentrated sludge and this seems to give you better

operation all the way around.

The Sparjers in the aeration tanks are supposed to be 7/32 inch in diameter but at the main plant they were made 9/32 inch in diameter by error. We had a lot of clogging of the Sparjers, so we ordered teflon inserts and we got 5/32 inch diameter openings and we noted a terrific increase in the back pressure on the eight-pound blowers. We found that when you increase the Sparjer openings from 7/32 inch to 9/32 inch there was no appreciable decrease in oxygen transfer efficiency and you get a lot more air and less back pressure on your equipment. Therefore, any plant that the OWRC may have with a small Sparjer opening, I wouldn't hesitate for them to enlarge it and use P.V.C. inserts, which are about a tenth of the cost of the teflon.

Another little operating gem, you may call it that, is when your operators take a settling test on your mixed liquor, of course with the step aeration process you should do it 40 to 50 feet pastthe entrance of your activated sludge; your five minute settling test should be about half of your 30 minute settling test. If you get that you'll know that it's going to settle out very well in the final settling tank. If you are not getting it, you are going to have bulking or rising sludge and you should investigate it.

Now with aeration tanks with spiral flow through them, you may have dead bands in the centre, so to me it seems a simple think to stick baffles intermittently along the length of the tank, just above the half-way point, and create more

mixing in there. All you have to do is build a couple of concrete columns and add some wood in between them. The baffling adds a lot of mixing action and nobody knows exactly how much more efficiency you will get out of the same amount of air output from a Sparjer or diffuser type of system. As a result, these things could be tried, if you are short of air, in your present plants. There probably are a few other points but I just can't think of them all at once and I certainly didn't come prepared to lecture on this. Now, I would just like to throw a question out here on the amount of agitation in aerators. Is it important and is there such a thing as too much agitation? I know that some paper claimed that you can have problems of shearing floc and trouble of settling in the final tanks, where others have claimed that you should get all the agitation you can. I would just like to know the experts' opinion on this point. Thank you.

A. Townshend

Could we have a reply from both Dr. Murphy and Mr. Crawford to that question.

Mr. Crawford

The question was concerning how violent can the mixing be without shearing the floc. We're talking about Sparjers which came along within the last ten years. From experience in four or five plants, we almost concluded that a minimum quantity of cfm of air per foot length of tank was critical. There also seemed to be some correlation of the mechanical aspects of the tank roll at Edmonton when they got down to about 7½ or 7 cfm per foot of

tank, irrespective of what the air required for the BOD, it seemed to be that the air requirements went up because the efficiency of the unit went down. In the Lakeview plant , we had Sparjers designed with about 5 or 6 cfm per foot. I thought perhaps we could put larger holes in the Sparjers at Lakeview, run all of the sewage through one of the aeration tanks, run them at a greater rate, and get a better efficiency. The tank was designed (we thought) for 2.5 MGD but was receiving 3.8 to 4 million gallons a day, with every bit as good an effluent as before. The air consumption per pound of BOD went down remarkably. Now whether that's over agitation or not I don't know; I'm not sure what the exact criterions are in the aeration tanks. Unfortunately, we haven't had these troubles in our aeration tanks at the Bay so we haven't had much experience on how to handle them.

Dr. Murphy

I will attempt to give a quick comment here. Your agitation is a direct function of the amount of energy which goes into the tank. If your air is the only source of energy then it's the quantity that is going to supply the amount of mixing. It doesn't make any difference if it's a fine or coarse bubble as far as the total energy in the tank goes. "Fair" at Harvard is currently doing some studies in which he is hoping to correlate the settleability of floc to the velocity gradients induced in the tank through the introduction of air. This goes back to the studies he did with Stein on flocculation. He is hoping to do this with activated sludge.

What about over-agitation? Yes, you can over-agitate, I think. A certain pharmaceutical firm in Chicago was operating very nicely until it was decided to increase the plant capacity. They decided to double their solids loading in the aerators and I think they are up around 12,000 ppm solids at the moment. This required rather fancy aeration devices and they went to a turbine type aerator that is used in the biochemical fermentation industry. This supplied all of the air they needed, but where they used to have a nicely settling sludge, they now have the best foam fractionator unit in Chicago. They get more activated sludge overflow in foam than they get in settling even after a half hour's gentle agitation to try to flocculate the particles. So you can go to over-agitation of these micro-organisms where you get them so beat up and attached to bubbles that they rise. Providing you know that they are going to rise, it's no worse a headache than having them settle.

A. Townshend

Thank you, Dr. Murphy. I'm open to general questions.

Ron Gotts - Industrial Wastes Division

I seemed to have gleaned two points from the papers. One is the characteristics of sludges that you get differ from plant to plant and the second being that you can expect a more efficient operation of final settling tanks if their design would be characterist of the sludge in mind. Now my question is, can one predict the characteristics of sludge, either experimentally or otherwise from a combination of raw sewage,

analyses and a knowledge of the plants that you have built accurately enough to warrant individual design of clarifiers?

Dr. Murphy

The sewage itself is micro-organisms, by the time you get it into your final clarifier the sludge will vary depending upon the degree of aeration. Principally, I think, the degree of aeration and the amount of available food is more important. Therefore, I would say that you can't design accurately from an analysis of raw sewage.

A. Townshend

Any other questions?

Mr. T. Kruk - Humber Plant

You can always improve the sludge going to the finals. I think the process which I will describe is working well and is one means of improving settleability in the final tanks. Following laboratory studies, we first added 5 percent supernatant from the digesters at about 2¼ to 3 percent solids, rather heavy supernatant to activated sludge in the first pass. The overflow from the digestion tank flows into the pre-aerators for about three days pre-aeration. We find that we improve the nitrification as far as the effluent goes and we also get a better settleability in the final settling tanks. If you have any means of getting digested sludge, whether you can pre-aerate it or not, over to the first pass of your aeration tank, do so. You'll find the slight additional BOD won't matter and you should get a activated sludge from the digested sludge. As a matter of fact, this

is the way I start up a new aeration tank. It saves a lot of time.

F. Guillaume - Division of Research

Tonight our discussion has dealt mainly with the design of settling tanks for the conventional activated sludge process. Lately, I have been concerned with the extended aeration process particularly the oxidation ditch. I wondered how the gentlemen here tonight would feel about changes of design criteria for settling tanks for this particular modification of the activated sludge process. Recently, I saw an article by "Downing and others" on a survey of three, small, extended aeration plants in England where they discuss design criteria such as surface loadings and retention time. Tonight, we heard three hours retention and 800 U.S. gallons per day per square foot. In England, they have the same three hours, but 600 Imperial gallons per day per square foot of surface area. However, Downing suggests for extended aeration plants a reduction to 150 gallons per day per square foot on the design flow for surface loading and increased retention to about six hours. I also saw a literature reference to an oxidation plant in Germany with clarifier operating problems with a particular type of activated sludge in it. The retention time was in excess of 6½ hours and the overflow rate at that particular time was about 200 gallons per day per square foot. I just wondered if anyone of the speakers tonight could shed some light on this.

A. Townshend

If there anyone in the audience who would like to tackle that one?

Dr. Murphy

A lot of extended aeration systems are characterized by so-called pin-point floc. I doubt if any reasonable detention time is going to remove a large fraction of this. The English have used longer detention in their final clarifiers for a long time. This may be because of the characteristics of their waste or it may be just simply standard practice. I'm not that familiar with it at all. I would think though, that one could take a look at a sludge produced in any sort of an aeration device, do a settling test on it, measure the suspended solids in the effluent after the settling test has been performed, and one would have some idea of the carryover solids from a perfectly built, ideally operating settling tank. This is going to be the optimum carryover you are going to get no matter what you do. I think this may vary with the concentration of sludge coming in and, consequently, I think the amount remaining in the supernatant will change as well. Nothing in the literature will indicate it will, but I think that you could reduce it down so that you would start getting quite a bit remaining in the supernatant. The initial settling rate of the interface when it forms will give you an idea of the sort of overflow rate you will have to use in an ideal tank. Now, then throw in a factor of safety to take care of your test cylinder and hydraulics and you may be in the right ball park. Whether this is 600, 800, 900 or 1200 I don't think you know until you actually take a look at the particular sludge. Certainly, this five-minute or 30-minute settling test is an interesting thing. Five minutes

is measuring, if you like, the clarification rate, while 30 minutes you are beginning to measure compaction. What you are doing is just judging a clarification rate of the mixture. I think it's a very good test and will probably tell you an awful lot about the initial settling of the sludge, which is what we are interested in as long as we are just talking separation and not talking underflow.

A. Townshend

I have one comment further to what Frank was asking and perhaps Doctor Murphy could answer it. May be with the oxidation ditch there is a certain amount of algae growth in with the activated sludge which we are not accustomed to in a conventional plant. Perhaps we are talking about a clarifier that must take out both algae and activated sludge.

Dr. Murphy

In my grand tour of the continent, I stopped in to see what McKinney was doing at the University of Kansas. He is developing what he calls an activated algae system. In this activated algae system - what he is going to do is grow algae to remove nutrients. He says it works in the lab. He tried it in the field with a pilot plant and it isn't working, but they are going to try it again. The interesting point is that he feels that he can remove algae just as easily as bacteria. It is all a matter, if I can quote him, "of where they are in their growth cycle."

Frank Guillaume - Division of Research

I left out two maybe important details in my earlier

question. One being that the mixed liquor suspended solids is generally up around 4,000 to 6,000 with any where from 80 to 95% settleable solids after 30 minutes. Return rates are generally close to 100%. The second point is that Dr. Murphy referred to compaction in the 30-minute settling test. I wonder should we modify our standard 30-minute settling test to include this theory and thereby come up with different values for the sludge volume index.

A. Townshend

Dr. Murphy, would you care to reply to that one?

Dr. Murphy

You would certainly get a different answer. For the work we've done or seen published, the answer is usually up by about a factor of two with any moderate amount of stirring. With much faster clarification and must faster compaction you are going to get entirely different results. It is just a matter of re-interpreting your results. Whether you want to do it in a standard test or not I don't know. If you want to change the results of your test and then try to apply them to the clarifier you have got to have some stirring. If you want to just have it as a standard test, then there is no point in adding the stirring. Actually, the stuff goes down so rapidly that it is pretty hard to distinguish what is happening.

A. Townshend

Are there any Commission staff here who can relate to us any specific problems that they have had to deal with in

connection with final clarifiers? - Either Sanitary Engineering or Plant Operations?

D. McTavish - Division of Plant Operations

I'm not sure that some of the problems we have had on final clarifiers are associated only with the final clarifier. Pin-point floc has been mentioned and we get this at a number of plants - some at only certain times of the year. One of the other problems that we have at some of the plants is not in the final tank but in the primary tank and that with return waste activated sludge. In a plant where the raw sewage is rather strong (high BOD) we do have problems in settling when we are trying to waste sludge to the primary clarifier. I don't know as we have any other specific problems. In Waterloo, a very overloaded plant, on occasion we have had a sludge that is close to bulking - you can see it almost up to the weirs in the final clarifier, but we get a very clear supernatant and very good effluent. If we are able to maintain the sludge at this point, we seem to be able to get very good effluent.

Jim McLellan - Division of Sanitary Engineering

We have been talking about rectangular settling tanks. George Crawford said that Gore & Storrie prefer these. I am wondering about square and round clarifiers - how the inlet end of these works will affect settling. Another thing is the difference in elevation between aeration tanks and your final settling tanks. When you discharge wastes or mixed liquor from the aeration tank into the centre of a round or square final settling tank, are you going to get more density

currents then if you have a smaller difference in elevation?

George Crawford

I don't think we said that we preferred rectangular tanks; we actually said our experience was more with rectangular tanks. I think we said that you could no doubt design just as successful a circular tank as a rectangular tank. With respect to the difference in elevation, I don't think there are any deleterious effects which we can complain about.

Jim McLellan

I'm thinking in particular of the Aurora plant which has an elevation difference of about a foot. Mixed liquor discharges into a centre hopper which is about four feet in diameter and extends down about three feet. Sludge coming into this hopper is going under rather than over and the hopper acts as a baffle really. This sludge, no matter what the suspended solids are in the aeration tanks or how fast you return, flows in a swath over the weirs. Now is this a function of the inlet condition or not?

Dr. Murphy

We did notice, in hydraulic testing, that the degree of short circuiting is increased when the velocity entering the tank increased. So the faster the waste enters the tank, the higher the degree of short circuiting in the tank.

John Thon - Research Division

Final settling tanks with gravity sludge returns are fairly popular with small extended aeration plants. My personal experience with those settling tanks is that you get very poor

performance. On some occasions, you get turbulence carryover into the settling tank and another frequent problem is sludge accumulation on the sides which eventually comes up to the surface and causes a nuisance. I was wondering if Dr. Murphy has had experience with these and would care to comment on it.

Dr. Murphy

You are talking about McKinney's favourite design. McKinney has long said that to build a good aerator you want perfect mixing. In any systems I've seen, I think you do get sludge stickage on the sides and if it sticks long enough, it is going to gasify and it's going to rise. You may get carryover or you may not.

A. Townshend

I would like to conclude our seminar this evening by giving very sincere thanks and appreciation to our three guest speakers and also to our guests in Metropolitan Toronto who took part in our discussion. Would you be good enough to acknowledge your appreciation (Applause). Meeting is adjourned.